Direct displacement based design of regular steel moment resisting frames

A. Jalali & M. Pourbaba & A. Kiani
Structural Engineering Department, School of Civil Engineering, University of Tabriz, Tabriz, Iran
Islamic Azad University of Maraghe, Iran
Shibrah Consulting Engineers, Tabriz, Iran

ABSTRACT: Displacement based design method represents a new approach to performance-based design. This research tries to assess direct displacement based design (DDBD) method for regular steel moment resisting frames and to develop reliable design method for them so that they withstand various seismic levels within certain performance levels. For this purpose, regular steel frames with 4, 8, 12, 16 stories are designed based on DDBD approach utilizing 2800 Iranian earthquake code, displacement spectrum. Finally, structures that are designed based on DDBD method were analyzed using nonlinear time history analysis under different ground motion records and acceptable results were achieved. According to the given results, drift that is proportional to damage, in most cases was less than assumed allowable drift and was acceptable. Maximum displacement Profile in the height of structure is also completely matched with the primary assumed design profile. Designed structures, have mostly experienced similar residual drift under different records.

1 INTRODUCTION

Direct displacement method is one of the most recently methods proposed for performance-based design of structures. During last decade, different methods have been proposed based on displacement-based design of structures but only a few are appropriately applicable within modern design codes.

Current investigation has been concentrated on direct displacement based design of regular steel moment resisting structures. To estimate nonlinear response of damped equivalent elastic model, here we have been used substituted structure concept of Gulkan & Sozen (1974) that has been recently developed by Priestley & Kowalski (2000) for direct displacement based design of regular and ductile RC structures. Current paper tries to propose reliable design procedures for seismic design of regular steel resisting structures within certain performance levels.

For this, here we have designed four 2D regular steel frames with 4, 8, 12, 16 stories based on DDBD approach, and utilizing 2800 Iranian earthquake code displacement design spectrum. Assuming that only beams are to be yield through estimating story yield drift, we have been used capacity design method to design frame elements under lateral loads. Finally, applying different design spectrum compatible ground motions we have performed nonlinear time history analysis for designed structures under DDBD methods. Given results are almost accurately meet seismic provisions of current codes. DDBD method is expected to be an alternative for current steel moment resisting frames design methods.

2 DDBD OF REGULAR STEEL MOMENT RESISTING FRAMES METHODOLOGY

2.1 *Displacement based design basis*

As expressed by Priestly (2003) there is credible evidence that one can relate the damage limit with strain which then these strains can be transformed into equivalent displacements. But it is not practical to relate the damage limit to force-level relationships directly.

In DDBD methodology, the original MDOF structure is substituted with an equivalent SDOF system. This equivalent system is represented by a secant stiffness K_e at maximum displacement Δ_d and an equivalent viscous damping including both the viscous and hysteretic damping of structure.

With the design displacement Δ_d determined (Eq. 14), and the damping estimated from the expected ductility demand ξ_{eq} (Eq. 19), the effective period T_e at maximum displacement response can be read from a set of design displacement spectrum. Representing the structure as an equivalent SDOF oscillator stiffness K_e at maximum response spectrum displacement can be found by inverting the equation for natural period of the SDOF oscillator namely:

$$T_e = 2\pi\sqrt{\frac{m_e}{k_e}} \tag{1}$$

$$K_e = 4\pi^2 m_e / T_e^2 \tag{2}$$

where m_e and K_e are respectively effective mass and effective stiffness of SDOF structure.

The design base shear V_B, at maximum response can be expressed as below:

$$V_B = F_u = K_e.\Delta_d \tag{3}$$

Determined base shear in accordance with the Eq. 3 is vertically distributed in proportion to vertical mass and displacement profiles. Thus:

$$F_i = V_B \frac{m_i \Delta_i}{\sum_{i=1}^{n}(m_i \Delta_i)} \tag{4}$$

where m_i, Δ_i are respectively related mass and design displacement at different storey's (i).

2.2 *Determination of design displacement Δ_d*

In many cases, the design displacement will be dictated by code drift limit (2~2.5 % of drift related to life safety performance level). However, generally maximum design drift θ_d, can be expressed as:

$$\theta_d = \theta_y + \theta_p \le \theta_c \tag{5}$$

where θ_y, θ_p, θ_c are respectively intrastory yield drift, plastic drift and code proposed drift.

Priestly has proposed equation below to determine yield drift of steel frames, θ_y:

$$\theta_y = 0.6\varepsilon_y l_b / h_b \tag{6}$$

where ε_y is the steel yield strain (ε_y=F_y/E), l_b is the beam length, F_y is the steel yield strength, E Young's modulus and h_b is the beam height.

As Gupta & Krawinkler (2002) have proposed, yield drift of steel moment resisting structures can be expressed as:

$$\theta_y = (\theta_{yb} + \theta_{yc}) = \left[\frac{M_{pb} l_b}{6EI_b} + \frac{M_{pb} H}{6EI_c}\right] \tag{7}$$

where M_{pb} is the plastic moment of beam, H is the story height, I is the cross-sectional moment of inertia, subscript b denotes a beam, and subscript c denotes a column. Plastic moment of the beams is obtained from:

$$M_{pb} = Z_b F_y = 1.14 S_b F_y \le \theta_c \tag{8}$$

where S_b is the section modulus. Substituting I_b=$S_b h_b/2$, we get:

$$\theta_{yb} = \frac{1.14 F_y l_b}{3Eh_b} = 0.38\frac{F_y l_b}{Eh_b} \tag{9}$$

The relative contribution of column to story drift will vary over a range of values depending on the values of I_c/H and I_b/l_b. Assuming that the contribution of column is 40% that of the beam we get:

$$\theta_y = 0.532\frac{F_y l_b}{Ed_b} \tag{10}$$

Plastic drift can be determined as below:

$$\theta_p = (\phi_m - \phi_y) l_p . \left[\frac{l_c}{l_b}\right] \tag{11}$$

where ϕ_m is critical curvature and l_c and l_b are the clear beam length between column faces and the beam length from column center to center respectively.

ϕ_y is the yield curvature proposed by priestly (2003) for steel sections as:

$$\phi_y = 2.10\varepsilon_y / h_b \tag{12}$$

2.3 *Determination of maximum displacement profile*

Kravasilis et al. (2006) using statistical analysis have been determined maximum displacement profile of regular steel moment resisting frames within elastic and inelastic ranges and in case of 3 column to beam capacity ratios (1.1, 1.3 and 1.5) and various story numbers.

$$\Delta_i = P_1.\theta_d.h_i.(1 - P_2.\frac{h_i}{H}) \tag{13}$$

The calculation of the parameters P_1 and P_2 is done with the aid of Table 1, as a function of the number of stories of the frame and the desired response range (elastic or inelastic). Please note that the three values of P_1 separated by a comma in the first column of the inelastic response case of Table 3, correspond to the three values of the joint capacity design factor α_{cd} (column to beam strength ratio), namely 1.1, 1.3 and 1.5.

Table 1. Values of the parameters of the proposed maximum displacement profile

Stories	Elastic response		Inelastic response	
	P1	P2	P1	P2
1	1.00	0.00	1.00	0.00
3	1.00	0.18	1.00	0.10
6	0.85	0.20	0.90	0.20
9	0.70	0.21	0.75, 0.80, 0.85	0.30
12	0.62	0.22	0.70, 0.75, 0.80	0.35
15	0.55	0.24	0.65, 0.70, 0.75	0.40
18	0.52	0.25	0.60, 0.65, 0.70	0.40

20	0.50	0.25	0.55, 0.60, 0.65	0.40

Having maximum displacement profile of stories determined, design displacement Δ_d, effective mass m_e and effective height h_e of equivalent SDOF system are as below:

$$\Delta_d = \sum_{i=1}^{n}\left(m_i\Delta_i^2\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right) \tag{14}$$

$$m_e = \sum_{i=1}^{n} m_i\Delta_i / \Delta_d \tag{15}$$

$$h_e = \sum_{i=1}^{n} m_i\Delta_i h_i / \sum_{i=1}^{n} m_i\Delta_i \tag{16}$$

In addition, design ductility is:

$$\mu_s = \frac{\Delta_d}{\Delta_y} \tag{17}$$

Design displacement and design yield displacement can be determined as below:

$$\Delta_y = 0.523\varepsilon_y\left(\frac{l_b}{h_b}\right)(h_e) \tag{18}$$

2.4 *Equivalent viscous damping*

Estimation of equivalent viscous damping factor (EVDF) is an important step in the methodology of the DDBD. Blandon (2005), For Ramberg-Osgood model (efficient for steel structure) the modified equivalent viscous damping factor for using in DDBD method is presented:

$$\xi_{eq} = \frac{125.25}{\pi}\left(1-\frac{1}{\mu^{0.45}}\right)\cdot\left(1+\frac{1}{(T_e+1)^4}\right) \tag{19}$$

2.5 *Design displacement spectra*

Since the structural period of the substitute structure is longer than that for the elastic structure (i.e. $T_e = \sqrt{\mu}T_i$, where T_i is the initial, elastic period), it is necessary for the displacement spectra to continue to longer periods than commonly plotted for acceleration spectra.

Displacement spectra for other than 5% damping have been determined using the European seismic code (EC8) modification factor of:

$$\Delta(T,\xi) = \Delta(T,5)\left(\frac{7}{2+\xi}\right)^{\frac{1}{2}} \tag{20}$$

2.6 *Building analysis for design moment*

In order to determine the design moments, the lateral force analysis of the structure should incorporate member stiffnesses representative of conditions at maximum displacement response. This is an essential component of the substitute structure approach (Shibata & Sozen, 1976). With a weak-beam/strong column design, beam members will be subjected to inelastic actions, and the appropriate stiffness will be:

$$I_b = \frac{I_b}{\mu} \tag{21}$$

Since the columns will be protected against inelastic action by capacity design procedures, their stiffness should be I_c, with no reduction for ductility.

3 DESIGN DETAILS

In this study, four two-dimensional steel frames with 4, 8, 12 and 16 stories have been designed using displacement response spectrum of 2800 seismic code (Fig. 1). All models have three bays with bay length of 5m and story height of 3m. Reference design acceleration assumed to be 0.5g and related drift of life safety performance level is considered 2.5%. In addition, assumed steel yield strength also is 2400 kg/cm^2.

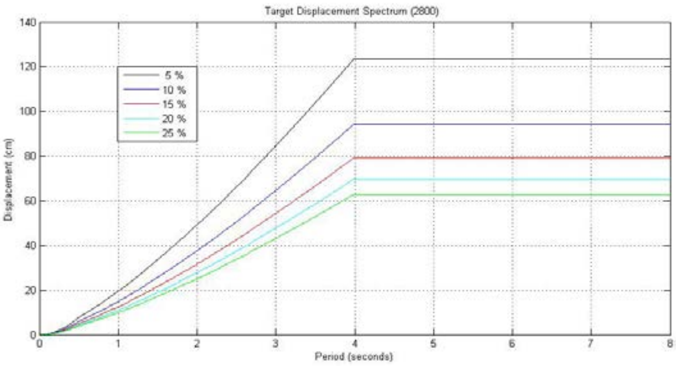


Figure 1. Design displacement spectra of Iranian 2800 seismic code for different damping ratios

Table 2, summarizes the Final design results of steel frames. It is also obvious that, based on DDBD philosophy, it is possible to design tall structures for a smaller base shear to weight ratio compared to short ones. It is noted that in the design procedure, column to beam strength ratio was evaluated carefully in order to assure the weak beam-strong column philosophy.

Table 2. Final design results of regular steel moment frames

Story	T	Δ_y	Δ_d	μ	ξ_{eq}	K_e	V_B	V_B/W
4	1.8	54.7	230	4.2	23.6	28	624	0.24
8	2.4	87.5	336	3.8	22.9	31	1003	0.19
12	2.7	131	396	3.0	20.4	36	1402	0.18
16	3.1	159	455	2.8	19.9	41	1805	0.17

*All dimensions in KN / mm

4 VERIFICATION OF THE DESIGN METHOD

In order to evaluate the seismic response of the designed structures, a series of non-linear time-history analyses under different 2800 code spectrum compatible records, have been performed. All non-linear analyses were carried out using fiber-element models developed in Seismostruct computer program.

Six near-field and far-field records of Table 3, have been utilized within time history analysis. All records were downloaded from http:\\www. peerberkeley.edu. (Peer strong ground motion database). We have tried to select records of same soil type letting shear wave velocity of soil be within ($375 \leq V_s \leq 750\ m/s^2$) range. In order to match records based on design spectrum we have used Rascal software, which has been designed utilizing random vibration theory and in addition to displacement domain, considers frequency content of records to match selected records within certain levels.

Figure 2 shows acceleration time-histories of one of these matched records.

Table 3. Characteristics of the selected records

Name	Year	Magnitude	Soil Type	Closest Distance	PGA
		(M_S)	(USGS)	(km)	(g)
Chichi	1992	7.62	C	31	0.36
Duzge	1999	7.30	B	2	0.19
Erzincan	1992	6.69	C	4.38	0.49
Imperial valley	1979	7.62	C	4	0.51
Kobe	1986	6.9	C	8.34	0.63
Tabas	1970	-	C	-	0.85

In DDBD method, displacement has been considered as the base of design methodology. Therefore, displacement parameters were selected as the controlling ones for performance assessment of the structures. In this regard, displacement time-histories, maximum story displacements, inter-story drifts and displacement ductility demands have been verified.

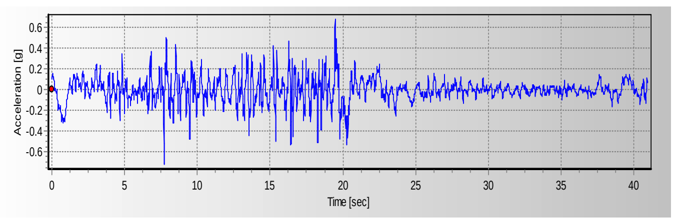
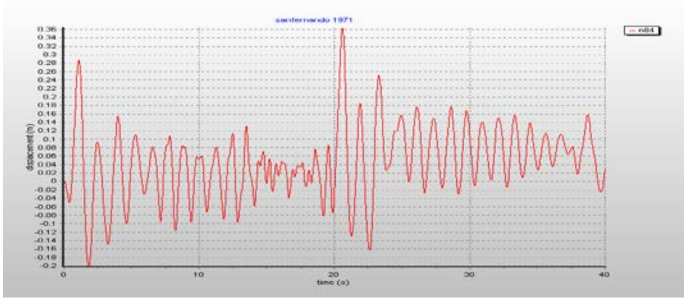


Figure 2. Acceleration time-history of Sanfernando record

Figure 3, shows a typical displacement time-history of 8-story frame under the Erzinjan record. Given results of time history displacement analysis under scaled records, always show a residual displacement



at the end of displacement time history.

Figure 3. Displacement response history for 8-story model under Sanfernando record

Figures 4 & 5 show maximum absolute story displacement profile of the frames under the six selected records and the design profile. The diagrams show a quite excellent performance of the method in limiting story displacements to the selected design value.

Inter-story drift is a very important verification parameter. Many studies (e.g. Priestley & Krawinkler) have shown that inter-story drift have a key role in damage potential of structures. Generally, model codes limit inter-story drift to values on the range of 2% to 2.5% of the story height. As mentioned earlier, a value of 2.5% was selected for this study. Figures 6 & 7 represent the inter-story drift profiles of all frames under different records. In these figures, the design inter-story drift profile is also displayed. Referring to these diagrams, the method performs quite satisfactorily. The shape of the profiles for tall models (12 & 16 story frames) are very similar to natural higher mode shapes of these structures derived from Eigen-value analysis of frames, implying that higher mode effects are important for tall frames

The last parameter which has been verified in this study is the story ductility demands. This parameter was calculated as the ratio of maximum inelastic story displacement and story yield displacement. The former was obtained directly from non-linear time history analyses, while the latter was calculated using Equation. 17. Story displacement ductility factors are shown in Figures 8 & 9. These figures show that a high degree of similarity exists between inter-story drift profiles and ductility demand profiles. Such similarity shows a direct relationship between displacement and ductility demands of the structures. Maximum story ductility demands occur when inter-story drift is at its maximum point.

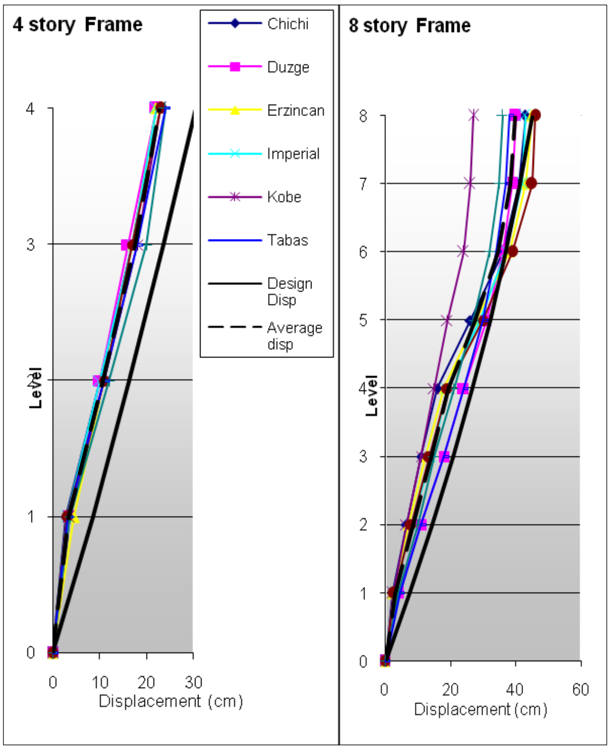


Figure 4. Absolute maximum story displacement (4 & 8 story model).

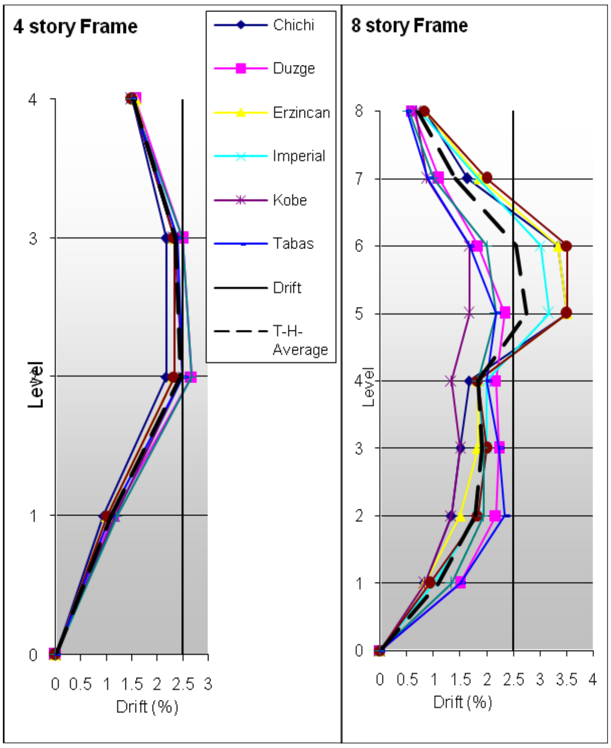


Figure 6. Maximum inter-story drift profile (4 & 8 story model).

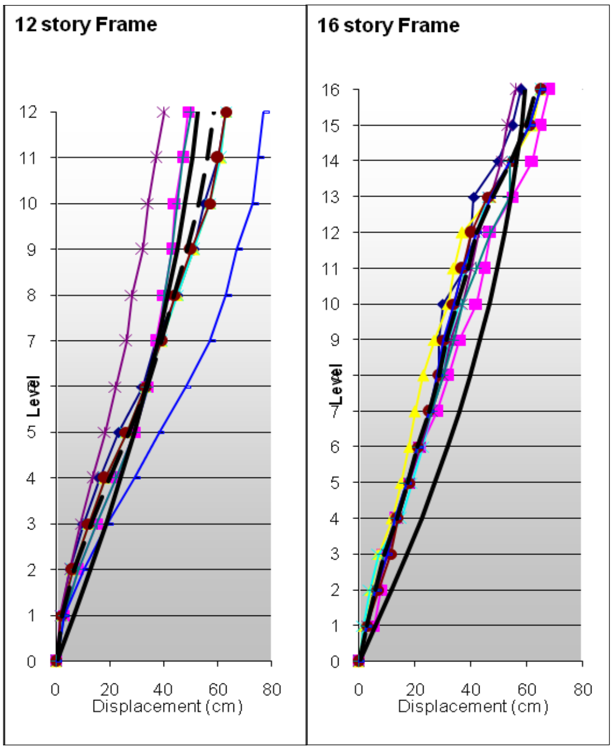


Figure 5. Absolute maximum story displacement (12 & 16 story model).

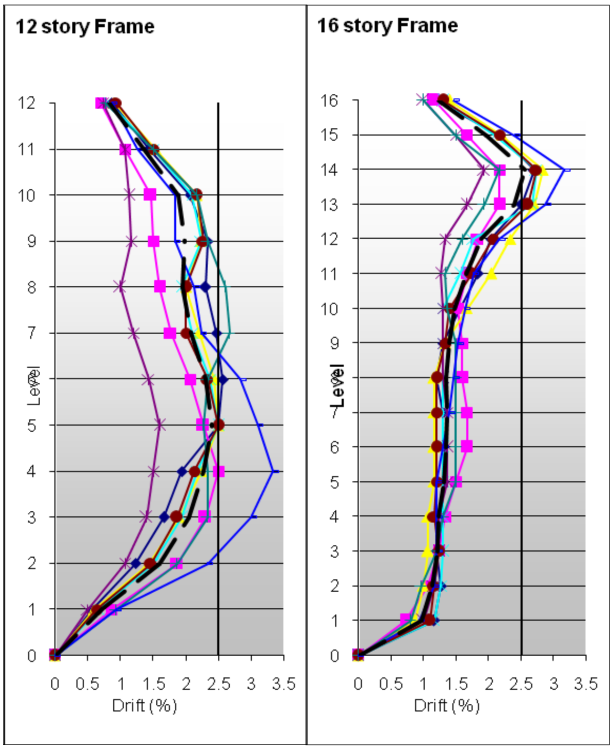


Figure 7. Maximum inter-story drift profile (12 & 16 story model).

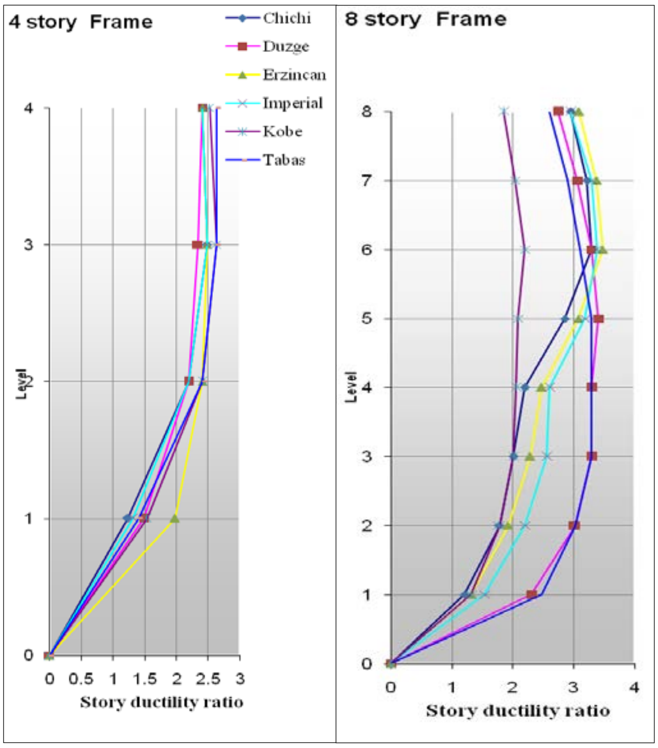


Figure 8. Story ductility demand profile (4 & 8 story model).

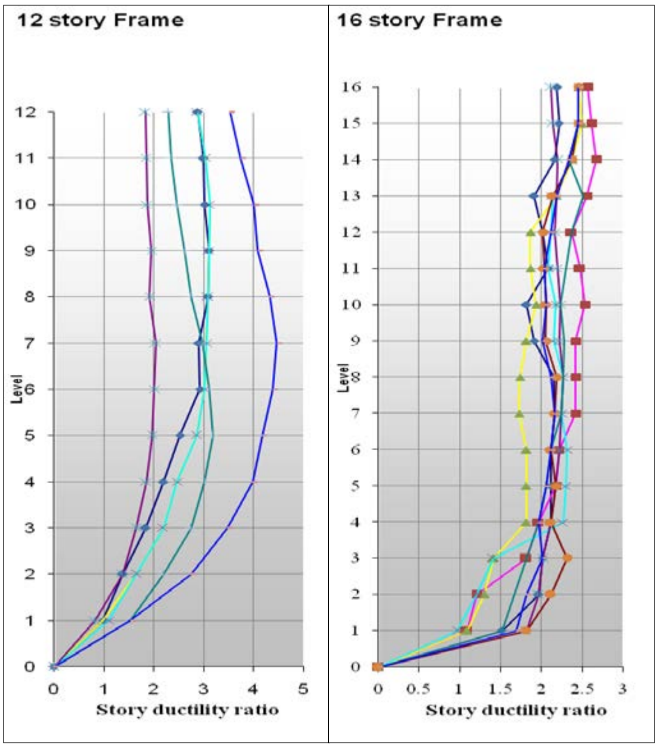


Figure 9. Story ductility demand profile (12 & 16 story model).

5 CONCLUSIONS

The present study focuses on seismic behavior of structures design with a new performance-based design tool called the direct displacement-based design. Performance verification studies show that the method can be regarded as an appropriate alternative to current erroneous force-based seismic design of structures.

The method, in terms of story maximum displacements, maximum inter-story drifts and story ductility demands performed quite satisfactorily, even for tall models.

The DDBD methodology is able to design structures with quite controlled residual behavior.

6 REFERENCES

Blandon, C.A. 2004. Equivalent Viscous Damping for DDBD. MSc Thesis, Rose School, Pavia Italy.

CEN, 1996. European Prestandard ENV 1998: Eurocode 8. Design provisions for earthquake resistance of structures. *Committee European de Normalization*, Brussels.

Gulkan, P. & Sozen M.A. 1974. Inelastic responses of reinforced concrete structures to earthquakes motions. *Proceedings of the* ACI 71 (12).

Gupta, A. & Krawinkler, H. 2002. Relating the seismic drift demands of SMRFs to element deformation demands. *Engineering Journal* 39(2), 100-108.

Karavasilis, T.L. & Bazeos, N. & Beskos, D.E. 2006. Maximum displacement profiles for the performance based seismic design of plane steel moment resisting frames. *Engineering Structures* 28 (2006) 9–22

Kiani, A. 2008. Direct displacement based design of regular steel moment resisting frames. *M.S Dissertation*, Tabriz University.

Paulay, T. 2002. Displacement Limits for Ductile ystems. *Journal of Earthquake Engineering and Structural Dynamics,* 31(2), 583-599.

Priestley, M.J.N. & Kowalski, M.J. 2000. Direct displacement-based design of concrete buildings. *Bull. New Zealand Nat. Soc. Earthq. Engrg.* 33(4).

Priestley, M.J.N. 2003. Myths and fallacies in earthquake engineering, revisited. *The Mallet Milne Lecture.* IUSS Press, Pavia, Italy.

Kin, W. Lee . 1985. Rascal Program: A Computer Program for Generated Synthetices Time History and Computes Peak Acceleration, Peak Velocity Response Spectral.

SeismoSoft. 2006. SeismoStruct- A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures", *Available online at http://www.seismosoft.com*

SEAOC. 1999. *Guidelines for performance-based seismic engineering*. SEAOC Blue Book - draft of Appendix I.